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Discussion of "HIGHWAY BRIDGES ON DEEP FOUNDATIONS"

by Louis Duclos
(Proc. Sep. 276)

ARTHUR V. WALKER,[2] A. M. ASCE.—The state highway system in Louisiana has been developed principally since 1923. During that time a considerable number of large bridges have been constructed under the supervision of the Department of Highways, State of Louisiana. These bridges are particularly notable because of the nature of the soil in which their foundations were constructed, which in many instances required the sinking of piers to very great depths. Louisiana is traversed by many large navigable rivers. The soil, particularly in the southern half of Louisiana, is alluvial to great depths and is plastic and readily subject to lateral flow (or displacement) when subjected to load. It is regrettable that the considerable accomplishments, developments, and experiences of the bridge engineers of the Department of Highways were not published previously for the benefit of other engineers who might be concerned with similar problems. Mr. Duclos was engaged in the design of many of the bridges which are mentioned in the paper, and he is deserving of commendation for an excellent coverage of the subject.

All the bridges cited by Mr. Duclos were partly financed through the Federal Aid program administered by the Bureau of Public Roads, United States Department of Commerce. The writer became familiar with the problem of deep bridge foundations while associated with the design and construction of the second bridge across the Red River at Moncla, and the bridge across the Calcasieu River at Lake Charles. He became very familiar with the condition and service records of the other bridges cited by Mr. Duclos (with the exception of the bridge at Miller's Bluff) by means of annual maintenance inspections of these bridges.

The depth of scour which can be expected to occur at bridge piers after their construction is of major concern in choosing the depth to which the piers should be constructed. It is to be expected that local scour will take place at bridge piers as a result of the turbulence in the stream created by the presence of the piers. This local scour can be observed readily at many bridge piers during periods of low water. The local scour which is visible at low water is, however, often not a true indication of the actual extent of the scour during periods of high water and increased stream velocities because the scour holes become partly filled with silt as stream levels recede and velocities decrease. A sounding taken in the relatively quiet water inside the fender of the pivot pier of the original movable bridge at Moncla during the flood of April, 1945, indicated that the scour at this pier had extended to an elevation that was approximately 40 ft below the low-water ground level at this point. It is true that this particular observation was the result of an unusual set of circumstances —a levee break located 3 miles downstream which caused high velocities at the bridge and a deflection of the stream current from its usual direction of flow by serious bank caving immediately upstream from the bridge. However,

[2] Div. Bridge Engr., Div. Ten, Bureau of Public Roads, U. S. Dept. of Commerce, Juneau, Alaska; formerly Dist. Bridge Engr., Bureau of Public Roads, U. S. Dept. of Commerce, Louisiana.

bridge washouts are usually caused by unusual circumstances. In 1953 soundings were made around old railroad bridge piers on the delta of the Copper River in Alaska during both high-water stage and average-water stage, and it was found that at two of the piers the scour holes had become silted after the recession of high water to the extent of 7 ft at one pier and 4 ft at the other. This silting action occurred during a drop in the river level of only 5 ft.

It is not always possible to anticipate the extent of erosion or bank caving which may take place at a bridge subsequent to its construction. However, if possible, the designer should locate the bridge piers so that they will not subsequently be subjected to large unbalanced lateral earth pressures because of erosion or bank caving which reasonably can be expected to occur. If bank caving is anticipated, the pier should be set well back from the river bank. In plastic soils, unbalanced lateral earth pressures can, over a period of years, result in a considerable leaning of bridge piers or abutments because of the increased footing pressures induced by these loads near the edges of the foundations. This movement of the bridge substructure often causes serious damage to the superstructure bearings and causes closure of, or damage to, deck expansion joints and adjacent parts of the bridge, particularly the concrete deck and curbs. Where substructure movement can be reasonably anticipated, it is well to provide generous openings at expansion joints to assure their continued functioning. During construction, the contractor should be required to distribute the material from the excavations in a manner so that avoidable unbalanced lateral earth pressures will not be induced against the substructure.

In the design of deep foundations, the designer should not be surprised to find that the usual relationships between foundation pressures (caused by dead load, live load, wind and longitudinal forces) in ordinary bridge substructures are much distorted. Where the piers are very deep and a considerable portion of their height is below any possible lower limit of scour, it is necessary to assume that horizontal loads of relatively short duration (such as wind loads and tractive loads) will be resisted by active horizontal earth reactions against the sides of the pier. To assume otherwise would require the bottom area of the pier footing to be so large as to be uneconomical and ridiculous. In checking the design of the bridge across the Calcasieu River at Lake Charles, it was found that the increased pressure on the foundations of the main river piers caused by the live load was only 4% of the total increased pressure resulting from both live load and dead load.

The interests of the bridge owner will best be served if the plans and specifications specially emphasize those results which are desired. Limitations and controls (other than those which are required by sound engineering principles) which place undue restrictions on the contractor's method of operation can only result in less than the maximum use of the contractor's knowledge, experience, and available equipment—with a consequent increase in the cost of the project. Contractors who engage in this type of construction usually have had a great deal of experience and have a better knowledge of the equipment available for the work and of the cost of performing the several detailed operations required in the construction than has the designer. Ac-

cordingly, maximum economy can be attained by allowing the contractor a wide choice of the methods of construction and specifying only the final results. Detailed plans for steel shells, fixed jets, and other details principally concerned with the method of operation could well be omitted from the plans and specifications and left to the choice of the contractor.

It would be of interest to examine plans for bridge piers of the type cited by the author. These would allow an engineer unfamiliar with this type of work to visualize the problem more clearly. Also of interest would be a tabulation of the unit friction values for the contact surface between the sides of the piers and the soil at various cutting-edge depths as actually experienced for the bridges cited. Information on the unit bearing values used under the pier footings for various types of soil would be informative as would a mention of the practice in Louisiana of expressing bearing values in terms of additional load over that load (caused by the weight of the superimposed earth) existing at the footing level before construction. The paper would be further enhanced by a more detailed presentation of the methods used and the sizes of charges employed in the use of dynamite inside the working chambers of the caissons when hard material was encountered.

Discussion of "PEAK DISCHARGE FOR HIGHWAY DRAINAGE DESIGN"

by Carl F. Izzard (Proc. Sep. 320)

MAX A. KOHLER,[14] A. M. ASCE.—The hydraulic design of highway structures is both an interesting and a timely subject. The importance of achieving an economical design is constantly increasing, and, as the time which can be justifiably devoted to the design of individual structures is limited, a generalized approach to the determination of the discharge to be used in the design is certainly useful and perhaps necessary.

[14] Chf. Research Hydrologist, Hydrologic Services Div., Weather Bureau, U. S. Dept. of Commerce, Washington, D. C.

Mr. Izzard has described methods for estimating the flood peaks having the specified recurrence interval from available discharge records. The work of Messrs. Dalrymple and Potter[2,5] on interpolation methods is cited by the author as well as the cooperative efforts of various state highway departments and the USGS in collecting stream-flow records from smaller watersheds. Mr. Izzard also makes mention of the feasibility of using USWB flood-forecasting techniques to extrapolate the stream-flow records.

[2] "Regional Flood Frequency," by Tate Dalrymple, *Research Report 11-B*, Highway Research Board, National Research Council, Washington, D. C., 1950.

[5] "Simplification of the Gumbel Method for Computing Probability Curves," by W. D. Potter, *SCS-TP-78*, Soil Conservation Service, USDA, Washington, D. C., May, 1949.

The USWB has developed precipitation-runoff relations and related procedures for estimating flood peaks over much of the United States.[15,16] Although errors in the computed flood peaks can be appreciable, the effect of such errors on derived frequency curves is not great if the errors are essentially random. In fact, pilot studies (yet to be reported on) indicate that the reliability of a frequency curve based on a 10-yr record can be enhanced greatly if the record is extrapolated four-fold using a "forecasting" procedure.

[15] "Predicting the Runoff from Storm Rainfall," by M. A. Kohler and R. K. Linsley, *Research Paper No. 34*, Weather Bureau, U. S. Dept. of Commerce, Washington, D. C., September, 1951.

[16] "Applied Hydrology," by R. K. Linsley, M. A. Kohler, and J. L. H. Paulhus, McGraw-Hill Book Co., Inc., New York, N. Y., 1949, pp. 630–635.

Although the development of reliable precipitation-runoff relations requires a minimum of from 8 yr to 10 yr of stream-flow records, such relations are usually applicable to all basins larger than a few square miles that are located within large, fairly homogeneous areas. Unit hydrographs and other techniques for distributing the computed runoff volumes can be developed from the records for only a few storms. It is obvious, therefore, that stream-flow data collected from small drainage areas can be applied long before 10 yr have elapsed.

In using forecasting procedures for this purpose, there are two related facts which must be considered. First, precipitation data (the length of the record and the number of stations) are frequently insufficient to augment appreciably the observed stream-flow record. Second, the analysis requires considerable time and thus may not be economically justified except for the design of the more costly projects or the development of a generalized approach. Both deficiencies can be minimized, however, if it can be shown that the precipitation

network utilized in the analysis need not be located in the basin under consideration, so long as the basin and the network are within an area of similar rainfall frequency regime. Most of the time required to estimate the maximum annual flood peaks for a period of years is spent in the processing of precipitation data. Thus, if the processed data can be used for numerous basins, the cost would become relatively small. It is believed that the area of applicability for a particular set of processed precipitation data could be determined by examination of intensity-duration-frequency maps. Because of the effect of topography on precipitation, it is obvious that rainfall data should not be transposed in mountainous terrain except with extreme caution.

It should be emphasized that this approach is a method of making full use of all available stream-flow data—and is not to be considered as a substitute for such data. Even so, the potentialities are believed to justify further investigation.

ROY F. WARNER,[17] M. ASCE.—Present (1954) knowledge, experience, and

[17] Hydr. Engr., Kansas City, Mo.

highway-drainage requirements have necessitated the new approach to the design of hydraulic structures which has been offered by Mr. Izzard. Much of the flood damage to highway structures has been the result of either a misapplication or an over-extrapolation of empirical formulas that were hardly more than "guessing rules" for a mode of transportation dependent on the unshod hoofs and wooden wheels of the nineteenth century.

The author has provided an excellent guide for the development of a detailed analysis of local runoff characteristics for any region in the United States. Fortunately, stream-gaging observations have been kept for a sufficient period of time to provide a fairly reliable experience record for frequency analysis. The convenient manner in which runoff frequency is presented makes it easy for the engineer to determine a reasonable design runoff. Thus, waste is prevented by overdesigning for abnormally high floods, yet the structure can be consistent with anticipated maximum conditions of service.

It would be helpful if runoff-frequency studies were made on typical watersheds varying in size from 2 sq miles to 200 sq miles. Within this size range, numerous factors—meander, channel storage, ground cover, soil characteristics, distribution of precipitation, and water shed geometrics—exert an influence on the runoff. Representative regions within major watersheds of each state should be studied to develop a basis for the adequate design of highway-drainage structures.

The cost of these studies would be minor compared to the half-billion dollars spent annually for the construction of highway bridges and culverts. A small fraction of 1% of the construction costs spent on frequency-runoff analysis of any state's major watersheds would result in enormous savings by avoiding overdesign of structures, by reducing reconstruction because of underdesign, and by avoiding or decreasing annual maintenance. The highway system in the United States is vast, but even so it is overtaxed by natural expansion and development. The system deserves to be constructed according to modern standards, with modern tools and by modern methods.

N. E. MINSHALL,[18] A. M. ASCE.—It is gratifying to note the progress that

[18] Hydr. Engr., Soil and Water Conservation Branch, Agri. Research Service, U. S. Dept. of Agri., Madison, Wis.

has been made in estimating peak rates of runoff for various frequencies on areas under measurement. The logical extension of these data to ungaged areas requires a thorough knowledge of the effect of many factors on rates of runoff. Mr. Izzard has drawn attention to various methods of estimating flood frequencies from gaged and ungaged areas. Two of the methods referred to by the author seem to be considerably at variance. Mr. Dalrymple plots the mean annual flood against the drainage areas as the basis for determining regional flood frequencies regardless of soil type, topography, shape of area, and vegetal cover.[2] Mr. Potter first outlines the general soil area and then attempts

[2] "Regional Flood Frequency," by Tate Dalrymple, *Research Report 11-B*, Highway Research Board, National Research Council, Washington, D. C., 1950.

to determine the effect of various other factors within this area by multiple correlation.[12] It is regrettable that, as soil groupings are available for most of

[12] "Rainfall and Topographic Factors that Affect Runoff," by W. D. Potter, *Transactions*, Am. Geophysical Union, Vol. 34, No. 1, February, 1953.

the United States, the data in all cases were not first grouped according to the major soil boundaries.

Relative to the effect of several of the principal topographic factors, Mr. Potter[12] states: "The index finally selected involves only the measurement of the length and slope of the principal waterway." This statement is essentially in agreement with that made[19] by Arnold B. Taylor and Harry E.

[19] "Unit-Hydrograph Lag and Peak Flow Related to Basin Characteristics," by Arnold B. Taylor and Harry E. Schwarz, *ibid.*, April, 1952.

Schwarz:

> "The study indicates that the most significant basin characteristics were drainage area, length of longest watercourse, length to center of area, and equivalent main stream slope."

In general, only the size of the drainage area has been considered in several of the flood-frequency reports cited by Mr. Izzard. Lack of adequate topographic maps was the cause of the failure to use other physiographic features in a multiple correlation.

It would seem more logical to group rainfall-runoff data by broad soil boundaries and then determine the cause for any unusual deviations from the mean line. For example, most of the stations shown in the claypan area of southeastern Kansas are near the upper line in Fig. 3. The fact that points 33, 35, and 37 are below this line is probably caused by the soils in this area being more permeable than those in the extreme southeastern part of Kansas.

On small watersheds the approximate peak rates of runoff (modified by factors for variations in land use and rainfall) have been shown by the single curve in Fig. 5. Any such simplification which can be made reasonably should meet with the approval of all persons responsible for the design of small drainage structures. There is, however, the danger that such widespread application will lead to serious errors.

Fig. 5 and the table of land-use factors assume that (*a*) the soil type is not an important factor in determining the peak rates of runoff from small watersheds, (*b*) the cover is effective in flood reduction regardless of the soil, and (*c*)

the slope of the land is the primary topographic influence, and little consideration need be given to the shape of the area or the slope of the main channel. The writer doubts that these exact limitations were intended by the author.

It is stated (under the heading, "Small Watersheds") that "The effect of soil type on peak rates of runoff has not been clearly established." The more impermeable the soil, however, the more frequently will it be in a saturated condition; thus the chance of heavy runoff occurring during an excessive rainfall is increased. The two maximum rates of runoff on the watersheds at Fennimore, Wis., in 16 yr have both occurred as a result of very intense storms on dry soil. Tests made by the SCS, with the type F infiltrometer,[20] have

[20] "A Comparison of the Colorado and Type-F Artificial Rainfall Applicators, Hastings, Nebraska," by L. L. Kelly, Office of Research, Soil Conservation Service, USDA, Washington, D. C., August, 1940 (mimeographed).

shown that the minimum infiltration rates on a number of moderately permeable soils of Iowa, Illinois, and Wisconsin varied from 0.4 in. per hr for corn fields to more than 1 in. per hr for good hay fields or pasture.[21] In contrast with this,

[21] "The Sny Watershed," Illinois Interim Survey Report, USDA, Washington, D. C., June, 1950, Appendixes, Fig. 15.

the minimum infiltration rates on the claypan areas of southern Illinois, Missouri, and southeastern Kansas are less than 0.1 in. per hr regardless of the type of vegetal cover. It is evident that the rainfall excesses are considerably higher on the impermeable soils.

The land-use factors given in the table in Fig. 5 for slopes greater than 2% might apply to the moderately permeable soils. It should be realized that the difference in infiltration rates between good and poor pasture may be as great as the difference between cultivated land and meadow land. Also, the engineer responsible for the design of small drainage structures has no assurance that a change in land ownership or other causes will not increase the severity of the cropping system. The engineer must provide for any such changes which might reasonably be expected to occur within the life of the structure.

Studies to determine rates and amounts of runoff from small agricultural watersheds were established by the SCS in 1938. Two groups of these watersheds located near Edwardsville, Ill., and Fennimore have been under the supervision of the writer since their inception. Reports—showing the general areas of application and estimated frequencies of peak rates of runoff—based on the first half of the period of record have been published.[22, 23]

[22] "Hydrologic Design of Farm Ponds and Rates of Runoff for Design of Conservation Structures in the Claypan Prairies," by D. B. Krimgold and N. E. Minshall, *Technical Publication No. 56*, Soil Conservation Service, USDA, Washington, D. C., May, 1945.

[23] "Rates of Runoff for the Design of Conservation Structures in the Upper Mississippi Valley Upland Loessial Areas," by N. E. Minshall, *Technical Publication No. 73*, Soil Conservation Service, USDA, Washington, D. C., January, 1949.

Ralph W. Baird[24] has shown that the effect of conservation practices or

[24] "The Effect of Conservation Practices on Peak Rates of Runoff," by Ralph W. Baird, *Texas Engineer*, Vol. 16, November 8, 1946, pp. 8–15.

crop cover on peak rates of runoff can be determined by comparing rates of rainfall minus runoff for the period of concentration for one watershed against another. Conservation practices on the base watershed must remain essentially unchanged for the entire period of record whereas the other watershed may have a few years of nonconservation practices followed by a period of conservation practices. This method of analysis was applied to the data obtained at Edwardsville and Fennimore, and the results are shown in Table 1.

TABLE 1.—PEAK RATES OF RUNOFF FOR VARIOUS FREQUENCIES[a]

Drainage area, in acres	Prevailing slopes, in percentages	Crop cover	Peak Rate of Runoff, in Cubic Feet per Second, to be Expected Once in			
			5 yr	10 yr	25 yr	50 yr
Claypan Prairies—Edwardsville, Ill.						
290	1 and 12	mixed[b]	480	625	800	930
50	1 and 12	pasture[c]	110	135	175	200
27	1	mixed	47	63	77	90
		cultivated[d]	51	67	81	95
		alfalfa[e]	43	59	73	86
Percentage reduction in runoff from 27-acre area by change from cultivafed to alfalfa			15.7	11.9	9.9	9.5
Upland Loessial Area—Fennimore, Wis.						
330	7	mixed[f]	315	425	595	660
171	6	mixed[g]	190	250	325	385
53	8	mixed	55	75	95	110
23	6	mixed	31	44	60	72
		60% cultivated[h]	37	50	67	80
		less than 10% cultivated[i]	20	31	44	54
Percentage reduction in runoff from 23-acre area by change from 60% cultivated to 10% cultivated			46	38	34	33

[a] On SCS watersheds. Based on period from 1938 to 1953. [b] Between 15% and 30% cultivated. [c] Approximately 100% for entire period of record. [d] 96% during 1938, 1948, and 1949. [e] 100% continuously from 1940 to 1943, inclusive. [f] Between 35% and 50% cultivated. [g] Between 45% and 60% cultivated. [h] During 1944 and 1945, less than 25% for all other years of record. [i] 1938, 1942, and 1947 through 1951.

The 27-acre area near Edwardsville was 96% cultivated during the years 1938, 1948, and 1949 and was 100% alfalfa continuously from 1940 to 1943, inclusive. The 50-acre area was approximately 100% pasture for the entire period of record, and the 290-acre area varied between 15% and 30% cultivated. Table 1 shows that peak rates of runoff on a 27-acre area of these claypan soils were reduced by approximately 10% for a complete change in cover from 100% cultivated to 100% alfalfa.

The 23-acre watershed at Fennimore was 60% cultivated during 1944 and 1945 and was less than 25% cultivated for all other years. The area had less than 10% cultivation during runoff-producing periods in 1938, 1942, and from 1947 through 1951. During these same periods, cover on the 171-acre area varied between 45% and 60% cultivated and, on the 330-acre watershed, varied between 35% and 50% cultivated. Table 1 indicates that peak rates of runoff on the 23-acre area of these moderately permeable soils were reduced by approximately one third when there was a reduction in percentage of area cultivated of from 60% to less than 10%. The percentage reductions in peak rates shown for these changes in vegetal cover are applicable only to the size of areas shown. The percentage reduction in rates of runoff, for a given change in percentage of cover, probably decreases as the size of the area increases.

Strict adherence to the curve in Fig. 5 may result in many inadequate designs, as is evidenced by the actual measured runoff rates and estimated

frequencies shown in Table 2. The 25-yr frequency from the curve in Fig. 5 for the 50-acre pasture area near Edwardsville is approximately half that determined from the actual records. The runoff of 80 cu ft per sec for this area has been exceeded on the average of once every 2 yr. On the 290-acre area the estimated runoff of 420 cu ft per sec has been exceeded on the average of once every 3 yr. The 4-yr record for the Little Sioux watersheds in western Iowa is insufficient for use in a frequency analysis. The actual measured values for these areas do show that use of values from the curve in Fig. 5 can be dangerously low for this area. It is possible that the three floods shown on the 309-acre watershed all greatly exceeded the suggested 25-yr frequency.

TABLE 2.—Comparison of the 25-Yr Estimated Peak Rates of Runoff with Actual Maximum Measured Discharges

Years of record	Drainage area, in acres	Type of cover	Type of soil	Peak Rate of Runoff in Cubic Feet per Second from			
				Fig. 5	Watershed record	Actual Measurement (Maximum Flood) During Period of Record	
				25-yr frequency		Date	Amount
Fennimore, Wis.							
16	23	mixed	mpsl[a]	63	60	June 28, 1945	62
	53	mixed		112	95	June 28, 1945	87
	171	mixed		235	325	Aug. 5, 1951	304
	330	mixed		370	595	Aug. 5, 1951	565
Ralston Creek, Iowa							
28	1,926	mixed and cultivated[b]	mpsl[a]	1,260	1,370	July 1, 1950	1,510
Edwardsville, Ill.							
16	27	mixed	claypan	88	77	March 30, 1938	70
	50	pasture		80	175	Aug. 14, 1946	150
	290	mixed		420	800	Aug. 14, 1946	825
McCredie, Mo.							
11	155	pasture	claypan	165	300	Oct. 3–4, 1941	265
Little Sioux, Iowa[c]							
4	63	mixed	mpsl[a]	125	...	June 15, 1950	315
						July 2, 1951	315
4	151	mixed		250	...	June 15, 1950	680
						July 2, 1951	400
						June 15, 1950	1,750
4	309	mixed		360	...	June 17, 1950	620
						July 2, 1951	950
3	49	cultivated		130	...	June 17, 1951	220
3	125	mixed		200	...	June 17, 1951	565

[a] The letters mpsl signify moderately permeable silt loam. [b] 35% cultivated. [c] Insufficient data for frequency analysis.

Crest-stage gages were established by the SCS in 1952 above twenty-two erosion-control structures in southwestern Wisconsin on drainage areas ranging in size from 250 acres to 2,500 acres. These gages show only the maximum peak rates between intervals of observation, but 10 yr, or more, of record will provide a reasonable approximation of flood frequencies. Because USGS topographic sheets are available for most of these areas, the data can also be of value in establishing the effect of various topographic features on the peak of runoff. Recording stations should be established on a number of areas to obtain the total seasonal and annual runoff for use as the basis for the design of small water-supply ponds and flood-retarding structures.

The writer doubts that reasonable accuracy in estimating peak rates of runoff from small ungaged drainage basins will be obtained until the results are grouped by soil types and an attempt is made to evaluate the effect of cover, land slope, shape of area, and slope of channel within each such area. Each soil group would have a curve similar to that shown in Fig. 5, but there would be additional factors if these curves showed a relationship for shape of area and slope of main channel. Sufficient data may not be available to establish such factors, but additional runoff-measuring installations should be made with this in mind.

CARL F. IZZARD,[25] A. M. ASCE.—The emphasis which Mr. Minshall places on the grouping of peak-discharge records according to soil type is fundamentally sound. This grouping should be the first step in attempting to discover the correlation of peak-runoff rate for a given frequency with other variables; otherwise a basic principle in statistical analysis—that samples be homogeneous—is violated. It is of interest to note that the USBPR has adopted the problem area map of the SCS as the criterion for geographic grouping of runoff data. This action stems from the highly significant correlations which have resulted from numerous applications of the criterion.

[25] Chf., Hydr. Branch, Div. of Research, Bureau of Public Roads, U. S. Dept. of Commerce, Washington, D. C.

The Allegheny-Cumberland Plateau studied by Mr. Potter[12] is one problem area (designated B-15); the glaciated shale and sandstone area of New York, Pennsylvania, and Ohio (B-8) is another. Charts for estimating peak discharge in both of these areas have been published.[26]

[26] "Use of Indices in Estimating Peak Rates of Runoff," by William D. Potter, *Public Roads*, April, 1954, pp. 1–8.

The problem area map should have been used in grouping the watersheds in Kansas which are designated by various symbols in Fig. 3; watershed 7 would then have been classified with those for various small rivers in eastern Kansas because the Walnut River Basin lies entirely within the Central Basin physiographic area. (Subdivisions of broad physiographic areas are called problem areas.) The boundary line between the Central Basin and the Great Plains is actually about 1° west of the arbitrary boundary placed at 96°30′ to classify stations plotted in Fig. 3. Similarly, point 5 for the Pawnee River would have been grouped with the points shown by the solid-circle symbol, all of which are

for watersheds wholly (or almost wholly) within the Great Plains area. The Blue River Basin straddles the boundary line between the Central Basin and the Great Plains areas and, quite reasonably, has a peak-discharge curve lying between the trend lines for those areas.

The problem areas were determined, as the name suggests, primarily to assist the engineers of the SCS to formulate land-use programs aimed at solving conservation problems. Consequently, factors such as soils, geologic formation, topography, and degree of erosion were all considered. As these factors also influence the rate of runoff, it is not surprising to find that the problem area map is also an excellent guide to the homogeneous classification of runoff records.

When the paper was prepared (1951), the application of statistical-correlation techniques to the analysis of peak-runoff data had not been fully explored. However, enough work had been done to prove that peak runoff for a given frequency did correlate satisfactorily with the size of the watershed—provided the data were all for the same geographic area. Now (1954) it is possible not only to define boundaries but also to explore, as Mr. Minshall suggests, the reasons why individual watersheds show runoff peaks deviating from the mean curve for a problem area. Mr. Potter and the USBPR have found that the differences in peak runoff between problem areas may be as great as tenfold, whereas the meteorological and topographic indices seldom explain differences greater than twofold.

Mr. Minshall, therefore, rightfully challenges the application of Fig. 5 without regard to the problem area within which a watershed may lie. Fig. 5 was prepared (1950) to provide an approximate method of estimating the peak rate of runoff by considering only the factors which could be shown to be significant from analyses then available. A method is now being developed which will include a geographic index by problem areas as well as other indices. This method will apply to watersheds of less than 10,000 acres.

Whereas land use, as demonstrated by Mr. Minshall's records, is a significant factor (especially on small watersheds tending to one type of use), it is being ignored in the current studies for two reasons: (1) The available runoff data (USGS records) do not cover enough range in land use within each problem area so that significant correlations can be made. (2) The complex of land use in a given problem area (for watersheds larger than field size) tends to be similar for the simple reason that the soil type and the related characteristics are also similar. The exceptions in most cases will be evident from examination of an aerial photograph. The engineer is expected to use his judgment in raising or lowering the peak-runoff rate obtained for the average condition. He will also need to anticipate changes in land use, especially where forested or pasture lands may be subdivided for housing developments. Data on the effects of changes in land use, such as those being accumulated by Mr. Minshall and the SCS, will be an invaluable guide to the engineer.

Mr. Kohler mentions the possible use of rainfall-runoff relationships to extrapolate short runoff records from longer rainfall records. Where the rainfall-runoff relationship in a given problem area has already been established (for flood-forecasting purposes), this method is probably practical. Further tests of its practicality should be made; they might result in a re-examination

of the rapidly expanding practice of installing crest-stage gages for obtaining records of peak runoff on small watersheds. Complete hydrographs are essential to the unit-hydrograph technique for estimating the peak discharge resulting from a given volume of excess rainfall. Synthetic unit hydrographs, as developed by Messrs. Taylor and Schwarz,[19] might have an application, but these also require complete hydrographs to establish the constants for a given problem area. However, if reliable results could be obtained with five years of record from continuous recording gages used in conjunction with a long period of rainfall record, the method suggested by Mr. Kohler might be more economical (and perhaps more reliable) than waiting ten or more years to get an adequate sample of annual floods from crest-stage gages.

Attention is directed to the fact that data relating rainfall intensity and duration for frequencies up to 10 yr are available for that part of the United States west of the 115th Meridian.[27] Such data can be used to establish rainfall indices for correlation with peak runoff in the part of the area not covered by Fig. 6. The other inadequacies of Fig. 6 are realized, and the USBPR is preparing, in cooperation with the USWB, rainfall index maps for the eastern half of the United States similar to those presented[26] by Mr. Potter.

[27] "Rainfall Intensities for Local Drainage Design in the United States—Part I West of the 115th Meridian," *Technical Paper No. 24*, Weather Bureau, U. S. Dept. of Commerce, Washington, D. C., August, 1953.

Research on peak rates of runoff—directed toward the highway engineer's problem—is now (1954) involving the close cooperation of many government agencies at both state and federal levels. Also noteworthy is the effort being made in these research studies to make maximum use of the hydrologic data which have been accumulating in reports for many years. Analyses of the existing data are pointing out the deficiencies in the basic data and will be invaluable in the formulation of a stream-gaging program to fill the gaps.

Discussion of
"A MATHEMATICAL EXAMINATION OF SPIRALED COMPOUND CURVES"

by T. F. Hickerson
(Proc. Sep. 357)

T. F. HICKERSON,[1] M. ASCE.—The author is grateful to Professor Hartman for his thorough investigation into the merits of the paper—a work that involved painstaking calculations on his part. Reinforced by this able discussion, the validity of the conclusions should become definitely established.

In the actual field layout of the spiral, the chord lengths must agree with the corresponding arcs along the length of the spiral, as pointed out by Professor Hartman. But in Example 4, where a 6° curve is connected with a 4° curve, there is no appreciable difference between chord and arc if 50-ft chords are used throughout. In fact, 50-ft chords may be used without correction even when the degree of curve extends up to 10°.

Attention is called to Eqs. (d) or (o) giving values of the angle β, a quantity that is needed in orienting the transit if a set-up at any intermediate point along the spiral is required. Thus in Example 4, let it be assumed that point A (C.S.) is at Sta. 72+60 and that the part of the spiral beyond Sta. 77 is to be run in by deflections from Sta. 77.

The forward deflection angle from Sta. 72+60 to Sta. 77 is

$$\frac{440}{200}\times 6^{\circ} - \left(\frac{440}{600}\right)^2 \times 120' = 13.2^{\circ} - 64.5' = 12^{\circ}07.5'.$$

Using Eq. (o), the value of β when applied to Sta. 77 is

$$\frac{440}{100}\times 6^{\circ} - \left(\frac{440}{600}\right)^2 \times 6^{\circ} = 23^{\circ}10.4'.$$

Now if the transit is set at Sta. 77, proceed as follows for orientation: Take a backsight to Sta. 72+60 with telescope inverted and vernier (on opposite side of 0°) reading $23^{\circ}10.4' - 12^{\circ}07.5' = 11^{\circ}02.9'$. Then plunge the telescope, unclamp the upper plate, and turn in azimuth until the reading is $0^{\circ}00'$, thus placing the line of sight on a forward tangent to the spiral at Sta. 77.

The degree of curve at Sta. 77 is $6^{\circ} - \frac{440}{600}\times 2^{\circ} = 4^{\circ}32.0'$.

$$\frac{1}{3}\ \theta_S = 2^{\circ} = 120'.$$

The deflection angles from Sta. 77 to the forward stations along the spiral are given in the table which follows:

1. Prof. of Applied Math., Univ. of North Carolina, Chapel Hill, N. C.

Transit at Sta. 77, vernier reading 0°00' on tangent.

Station	Deflections		
	Circle	Spiral	Difference
77+50	1°08.0'	0°00.8'	1°07.2'
78	2°16.0'	0°03.3'	2°12.7'
+50	3°24.0'	0°07.5'	3°16.5'
78+60(S.C.)	3°37.6'	0°08.5'	3°29.1'

The backward deflection from the S.C. to Sta. 77 is

$$\frac{160}{200}\times 4^{o} - \left(\frac{160}{600}\right)^{2}\times 120' = 3^{o}20.5'.$$

As a final check,

$$3^{o}29.1' + 3^{o}20.5' + \beta = 30^{o} = \Delta.$$

CORRECTIONS

(1) Page 357-1, 2nd line, the name should be T. F. Hickerson not H.F. Hickerson.

(2) Page 501-17, 3rd line, the name should not be T. F. Hickerson but should be Paul Hartman, A.M. ASCE.

(3) Page 501-18, Eq. (t) should read

$$\tan \Delta_2 = \frac{R_1 \sin \Delta - x_1}{R_1 \cos \Delta + y_1 - R_2}$$

Discussion of
"EXPEDIENT GEOMETRY FOR THE DESIGN
OF COMPOUND HIGHWAY CURVES WITH CONNECTION SPIRALS"

by Robert D. Schacherl and George J. Berindoague
(Proc. Sep. 500)

T. F. HICKERSON,[1] M. ASCE.—"That more than 3,000 curves with transition spirals were used on the 310 miles of highway from Cochabamba to Santa Cruz in Bolivia" is indeed a record not matched elsewhere in the world, most probably.

The offset p (which might refer to p_a, p_1, p_2) is evaluated by the formula:

$$p = L_s\left[\frac{\theta_s}{12} - \frac{\theta_s^3}{336} + \frac{\theta_s^5}{15840} - \cdots\cdots\right], \qquad (a)$$

where L_s (which might refer also to L_a, L_{s1}, or L_{s2}) is the length of spiral and θ_s (which might refer also to θ_a, θ_{s1}, or θ_{s2}) is the equivalent spiral angle in radians.

Eq. (a) shows that for any given value of θ_s, p is directly proportional to to L_s, meaning of course "p for a 400-ft spiral is 4 times the value of p for a 100-foot spiral."

Expressing θ_s in degrees and letting L_s = 100 feet, Eq. (a) becomes:

$$p = 0.1454441\theta_s - 1.582315\theta_s^3\times10^{-6} + 1.022426\theta_s\times10^{-11} - \cdots \qquad (b)$$

If $\theta_s = 20^\circ$, p= 2.90888-0.01266+0.000033 = 2.89625.

It is seen that the second and third terms of Eq. (b) are practically negligible when θ_s is less than about 20°. Hence for this range of values p is directly proportional to θ_s, $(D_{c2}-D_{c1})$, or D_{c2}, treated separately, if D is less than 25°, as stated by the authors.

As a substitute for the Bartlett formulas used by the authors in preparing GRAPHS, the writer proposes another more comprehensive set as follows:

$$T_2 = \frac{N_1}{\sin\Delta}, \qquad (c)$$

$$T_1 = B_1 - N_1\cot\Delta, \qquad (d)$$

where

$$N_1 = R_1 - (R_1 - R_2 - p)\cos\Delta_1 - R_2\cos\Delta, \qquad (e)$$

$$B_1 = (R_1 - R_2 - p)\sin\Delta_1 + R_2\sin\Delta, \qquad (f)$$

$$\Delta = \Delta_1 + \Delta_2. \qquad (g)$$

1. Prof. of Applied Math., Univ. of North Carolina, Chapel Hill, N. C.

Knowing that the forward and backward deflection angles from the CS and the SC (here called ϕ_1 and ϕ_2 respectively) are:

$$\phi_1 = \frac{L_s D_1}{200} + \frac{1}{3}\theta_s \; ; \quad \phi_2 = \frac{L_s D_2}{200} - \frac{1}{3}\theta_s \; ; \qquad (h)$$

one may use the check:

$$\phi_1 = \text{arc tan } \frac{N_1}{B_1} \qquad (i)$$

to catch a possible error in previous calculations.

Also,

$$c = \text{Long Chord} = \frac{B_1}{\cos \phi_1}. \qquad (j)$$

As a final check, one has

$$c = T_1 \cos \phi_1 + T_2 \cos \phi_2. \qquad (k)$$

Apropos of the author's reference to the "cubic spiral," the writer would point out what is doubtless well known to him, namely, that all transition spirals of the usual form (where curvature varies as the distance from the TS) are fundamentally the same whether they be called cubic spirals, railway spirals, or highway spirals. The alleged variations between types of spiral curves have been over emphasized in this country often to the confusion of students as well as men in practice.